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EVALUATION OF SEISMIC DAMAGEABILITY OF A TYPICAL R/C BUILDING IN MIDWEST UNITED STATES

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ABSTRACT

The seismic evaluation of a four story RC building is summarized. The building is representative of low rise RC buildings in the Memphis area. Nonlinear time-history analysis, as well as simplified elastic and inelastic response evaluation methods, are used to determine the response for five earthquake motion intensities. The expected damage is then quantified using the fatigue based damage model.

KEYWORDS

LAMB Project; Seismic Evaluation; Damage Quantification; Gravity Load Design Structure.

INTRODUCTION

The Loss Assessment of Memphis Buildings (LAMB) project is one of the major research projects funded and managed by the National Center for Earthquake Engineering Research. The immediate goal of the project is to asses probable losses and risk levels, for specific building types, in the Memphis area. However, the long term goal of the project is to develop a unified methodology that can be used for other building types, and other geographical regions. The project studies the response of non-ductile reinforced concrete buildings and unreinforced masonry buildings. The LAMB integrates research efforts in structures and systems, seismic hazard and ground motions, geotechnical engineering, risk and reliability, and socioeconomic aspects.

The present paper summarize the results from one of the building model development and analysis tasks of the project. The results presented provide key elements to verify and validate the models used to develop fragility curves, which are the link between different structural types and their loss and risk estimates, as a function of the maximum intensity of ground shaking. The structure presented is representative for low rise reinforced concrete buildings in the Memphis area.

In this paper, the performance of a four story building, designed without seismic provisions, is studied when subjected to different intensities of ground motion. The seismic demand in the structure is determined in two ways: (*i*) time-history analyses; and, (*ii*) using simplified elastic and inelastic response evaluation methods. The latter technique was recently developed by the authors (Reinhorn *et al.*, 1996).

The deformation demands imposed by the earthquakes are then compared to the ultimate deformation capacities, to translate response quantities to damage indices, and latter to damage states.

BUILDING DESCRIPTION

The structure described herein was selected as the typical low-rise reinforced concrete building for the Memphis area. The structure, part of the Memphis state university, was designed in the mid 1960's without seismic provision, as most buildings in the area. The four story reinforced concrete structure is used as a classroom building. Plan dimensions, typical at all floors, are 197 feet by 95 feet (see Fig. 1), with a typical story height of 12 feet. The structural system consists of a ribbed slab supported by columns and shear walls.

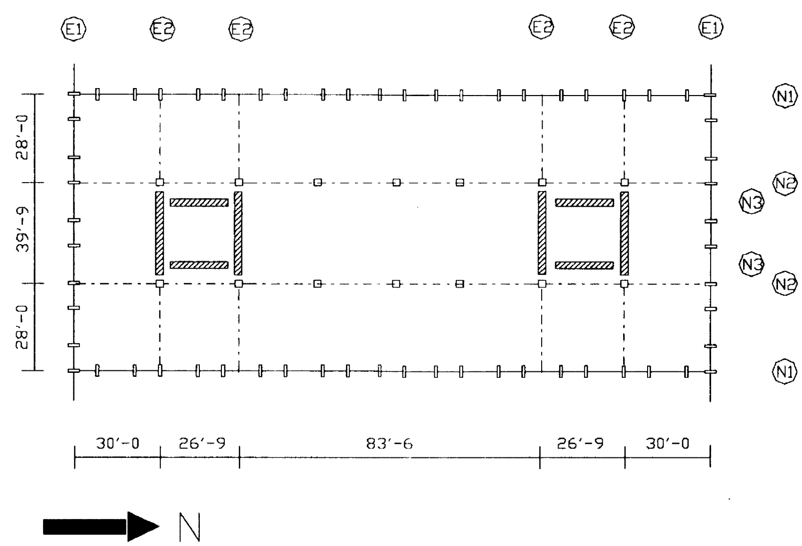

Fig. 1 Typical floor plan for case study building.

Typical column dimensions used in the building are 2'6" by 1' columns closely spaced in the perimeter of the building, and 18" by 18" in the interior. Similarly, two beam types can be identified: one for the exterior frames, and a second one for the interior frames. All beams, or "slab bands" as referred to in the initial design, are 17" deep. The shear walls, delimiting the elevator and stairs, have a constant thickness of 1' throughout the height of the building. Figure 2 shows an elevation of the three frame types identified in the longitudinal (North-South) direction. For a more detailed description of the building see Valles *et al.* (1996a).

To evaluate the seismic response of the building, the latest version of the computer program IDARC was used (Valles *et al.*, 1996b). A two dimensional model for the structure was used since no significant torsional amplifications are expected due to the symmetry of the building. Furthermore, only half or the building is modeled in either direction. Note that all results reported consider only half of the actual building. The story weights and story heights for half building are summarized in Table 1.

Table 1 Story heights and weights for the case study building

Story	Story Height (in)	Story Weight (kips)	Cum. Weight (kips)
4	144	1833	1833
3	144	2284	4117
2	144	2284	6401
1	144	3102	9503

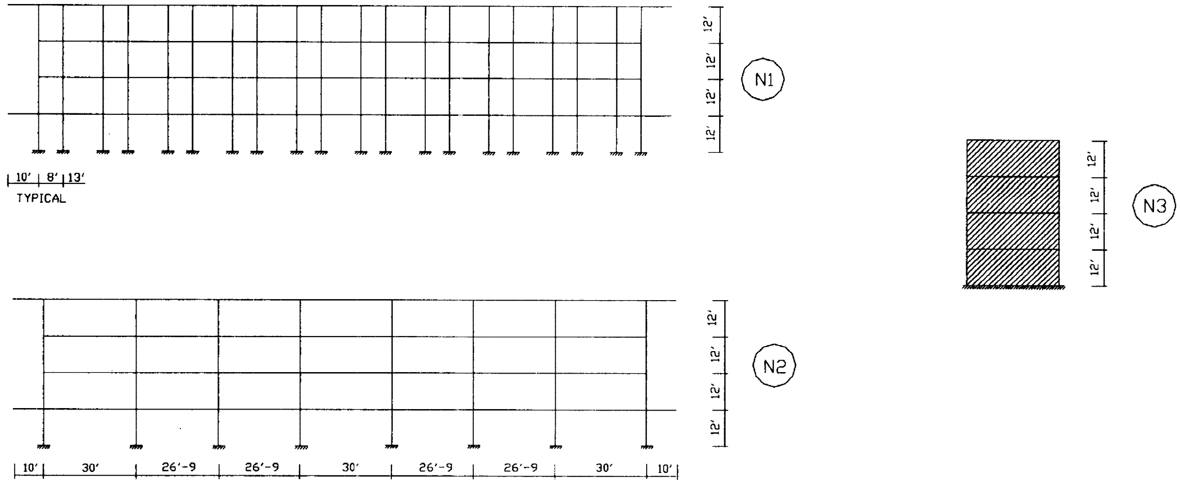

Fig. 2 Frame types in the NS direction.

Due to the higher stiffness of the shear walls, as compared to the column-beam frame action, the lateral loads in the system are mostly carried by them. Nevertheless, the model includes the other frames to ensure that the imposed displacement profile will not threaten the vertical load capacity of the structure. Since the limit state in the building can be reached either by a high force demand in the shear walls, or a significant displacement demand in the moment resisting frames, all structural elements were included. The moment-curvature capacity curves for each structural element was automatically generated by the program IDARC version 4.0 (Valles *et al.*, 1996b), using the fiber model capabilities.

The Memphis metropolitan area is located in the Mississippi embayment, near the epicenters of the New Madrid series of earthquakes (Nuttli, 1974). Attenuation relations for the area have been suggested by Nuttli and Hermann (1984). Ground motions for the site, corresponding to different earthquake magnitudes, stress drop, and epicentral distances, were generated by the seismic hazard and geotechnical task group of the LAMB project. The records were generated combining a deterministic and a probabilistic approach (Horton, 1994). A total of 200 records were generated for the area, out of which, five records were selected to be representative of five ground motion intensities, with peak ground accelerations (PGA) of 0.1g, 0.2g, 0.3g, 0.4g, and 0.5g.

TIME-HISTORY ANALYSIS RESPOSE

The structure was subjected to the five earthquake motion intensities selected. Figure 3 presents the maximum displacements, interstory drifts, and story shears experienced by the structure (NS direction), and for each earthquake intensity. The integration time step for the 0.4g and 0.5g intensities had to be considerably reduced, since the motion induced significant inelastic excursions in the structural elements.

PUSHOVER ANALYSIS

The nonlinear pushover analysis, or collapse mode analysis, is a simple and efficient technique to study the response of a building. The pushover analysis is carried out by incrementally applying lateral loads, or displacements to the structure. The sequence of component cracking, yielding, and failure, as well as the history of deformations and shears in the structure, can be traced as the lateral loads (or displacements) are monotonically increased (see Fig. 4). Furthermore, strength and service limit states, such as the failure of an element, the formation of a collapse mechanism, etc., can be identified.

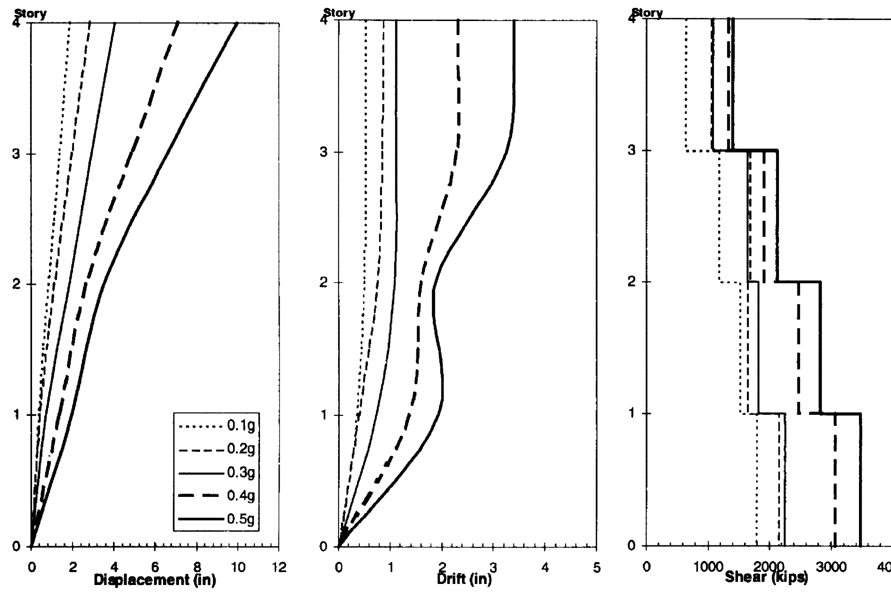

Fig. 3 Maximum response in the NS direction for the five earthquake motion intensities.

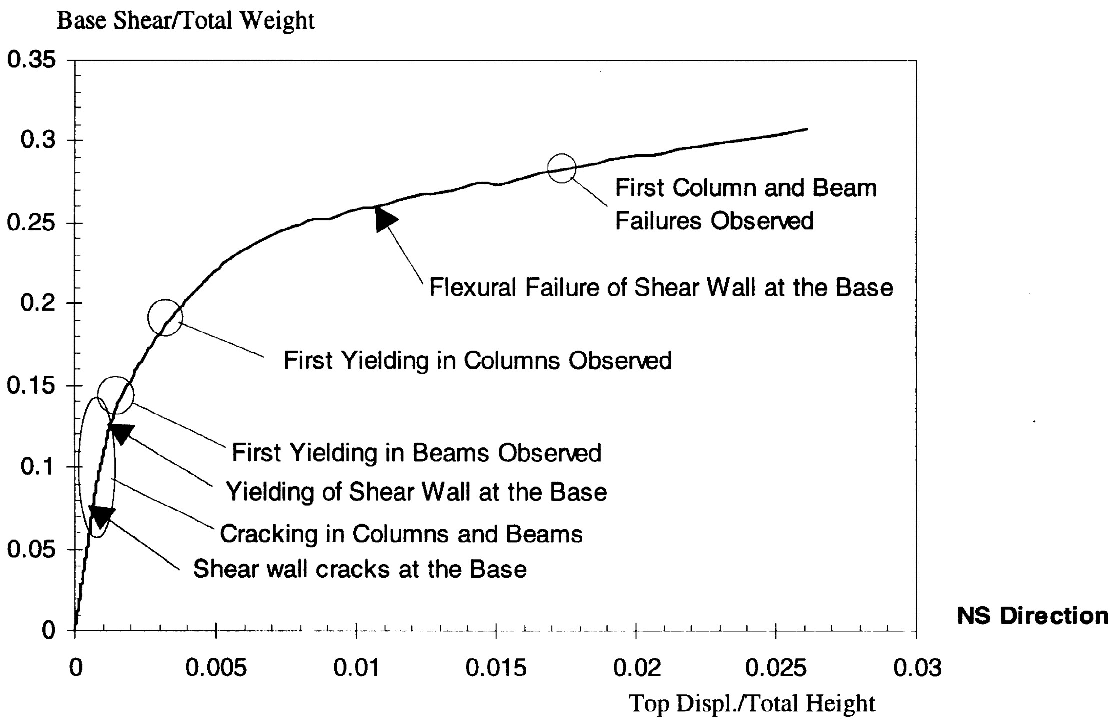

Fig. 4 Critical stages along the overall pushover capacity curve.

The results from pushover analyses are often presented in graphs that describe the variation of the story shear versus story drift, for an inter-story description of the capacity, and base shear versus top displacement, for a global description. The capacity curve determined from a pushover analysis is influenced by the lateral force (displacement) distribution used to load the structure. Figure 5 shows the overall capacity curves for a constant (k=0), linear (k=1), and parabolic (k=2) distribution. The first shear wall failure, as well as the response obtained from time-history analyses are sketched in the figure. The capacity curve for k=1 (linear) was found to capture the results from the time-history analyses in the deformation range before the first shear wall failure.

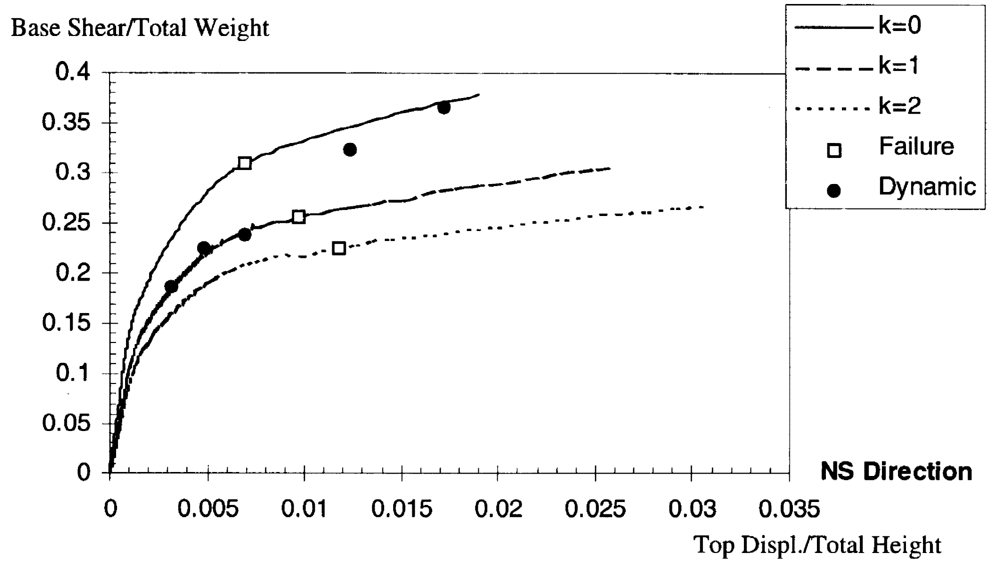

Fig. 5 Overall pushover curves and nonlinear dynamic analysis results.

SIMPLIFIED ELASTIC AND INELASTIC RESPONSE EVALUATIONS

In the design or evaluation process, the capacity of the structure is estimated, and compared to the demand loads. The simplified methods briefly described estimate the inelastic response of buildings, without performing the computationally demanding time history analysis, estimate the response as the intersection of a capacity and a demand curve. The method is based on the capacity spectrum method proposed by Freeman (1994). The force deformation capacity curves are determined from pushover analysis. The elastic demand curve, determined from an elastic spectral analysis, is modified to account for the hysteretic energy dissipation.

The elastic response evaluation method considers the use of an equivalent linear system to estimate the nonlinear response (see Fig. 6). A summary of different methods to determine the equivalent period and damping ratio is presented by Iwan and Gates (1979). The average stiffness and energy method seems to give the smallest percentage of error for various ductility ratios (Iwan and Gates, 1979). For this method, the equivalent period and critical damping ratios are defined according to:

$$T_{eq}/T_0 = \left[\frac{1-\alpha}{\mu}(1+\ln\mu)+\alpha\right]^{-1/2} \quad ; \text{ for } \mu > 1 \tag{1}$$

$$\xi_{eq} = \left(\frac{3}{2\pi\mu^2}\right)\frac{\pi\xi_0\left[(1-\alpha)\left(\mu^2-\frac{1}{3}\right)+\frac{2}{3}\alpha\mu^3\right]+2(1-\alpha)(\mu-1)^2}{(1-\alpha)(1+\ln\mu)+\alpha\mu}; \text{ for } \mu > 1 \tag{2}$$

The idealized bilinear pushover capacity curve is superimposed with the equivalent linear demand curves. The point where the ductility along the capacity curve coincides with the equivalent ductility of the intersecting demand curves, yields an estimate of the inelastic response.

For the inelastic response evaluation method, the spectral curves are generated using a bilinear model for the structure (Reinhorn *et al.*, 1996). The response of the bilinear single-degree-of-freedom system is obtained for a given value of the post-yielding stiffness (α), and for different values of the force reduction factor, defined as the ratio of the elastic to the yield force capacity of the system:

$$R = V_e / V_y \tag{3}$$

The point where the demand curve, corresponding to the actual value for R, intersects the capacity curve,

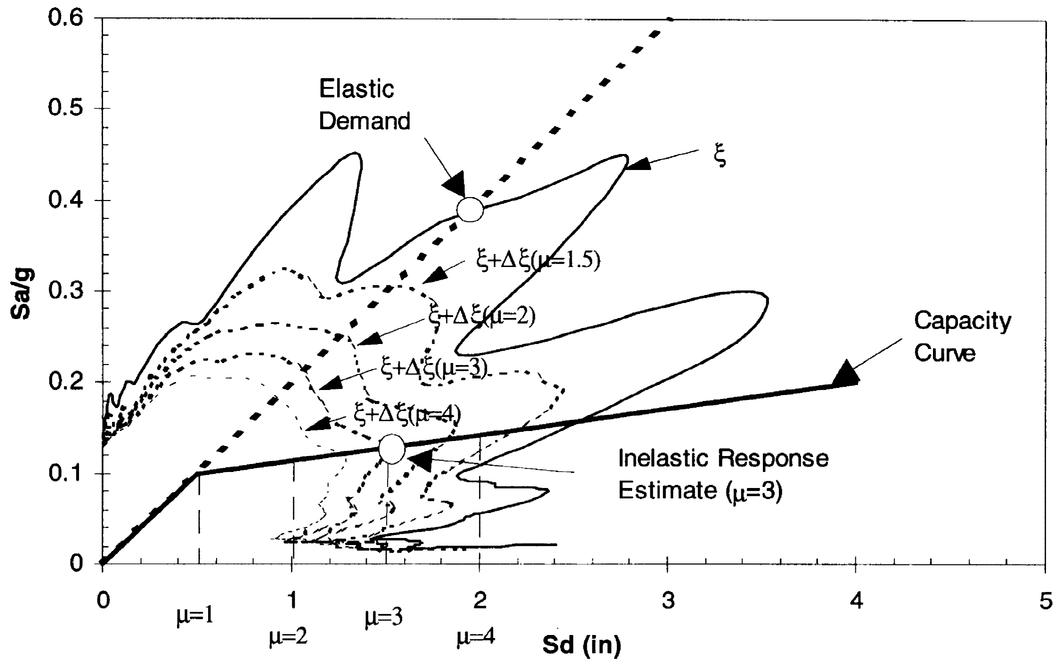

Fig. 6 Simplified elastic response evaluation method.

is the actual inelastic response of the bilinear structure (Fig. 7).

The procedure described for a sdof system can be extended to multi-story buildings by modifying the capacity curve to an equivalent sdof system. The response can be evaluated considering the overall building response, or the interstory response. In the former case, the top displacement versus base shear is used to characterize the capacity, while in the latter case, the story drift versus story shears are used. Both evaluation levels are important, since the overall response provides an estimate of the global performance, while the story evaluation will detect undesirable weak stories. Actual overall pushover results (δ_T and V_b) are modified to an equivalent sdof spectral pushover curve (S_d and S_a / g) according to:

$$\frac{S_a(T_j,\xi_j)}{g} = \frac{V_b}{g\Gamma_j^2}; \text{ and } S_d(T_j,\xi_j) = \frac{\delta_T}{\phi_{N,j}\Gamma_j} \tag{4}$$

that consider a single mode contribution. The formulas for multiple mode contribution, and for the interstory response evaluations, can be found in Valles *et al.* (1996a).

Figure 8 presents a comparison of the overall simplified response evaluations versus the dynamic analysis results. Note that the predictions agree fairly well except for the last two intensities, when the pushover curve for k=1 cannot capture the behavior. Using the pushover curve for k=2 would yield better estimates for the last two response quantities.

DAMAGE QUANTIFICATION

A damage index is a parameter that indicates how close the maximum response is to the maximum ultimate capacity of the structure. Often, damage index models are normalized from a value of "0", indicating negligible response quantities as compared to the ultimate capacity, to a value of "1", indicating that the ultimate capacity of the structure has been reached. The response quantities determined for the building are first used to calculate damage indices, which are then correlated to probable damage states. Several formulations have been proposed for the damage indices. The fatigue based damage model, suggested by Reinhorn and Valles (1995), was used in the study. The damage index is defined as:

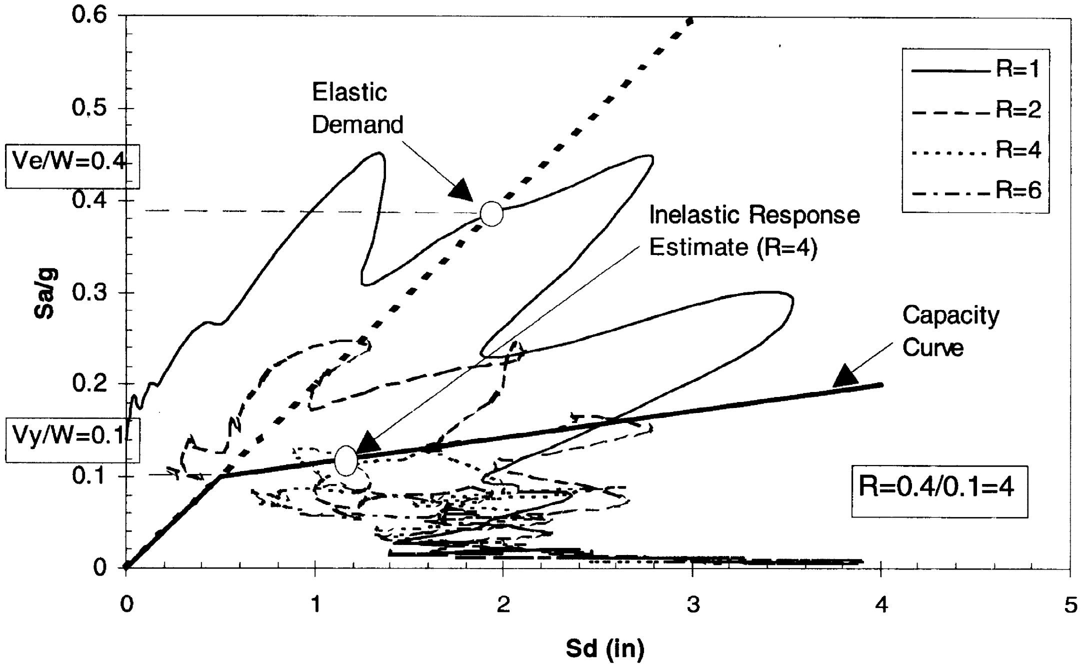

Fig. 7 Simplified inelastic response evaluation method.

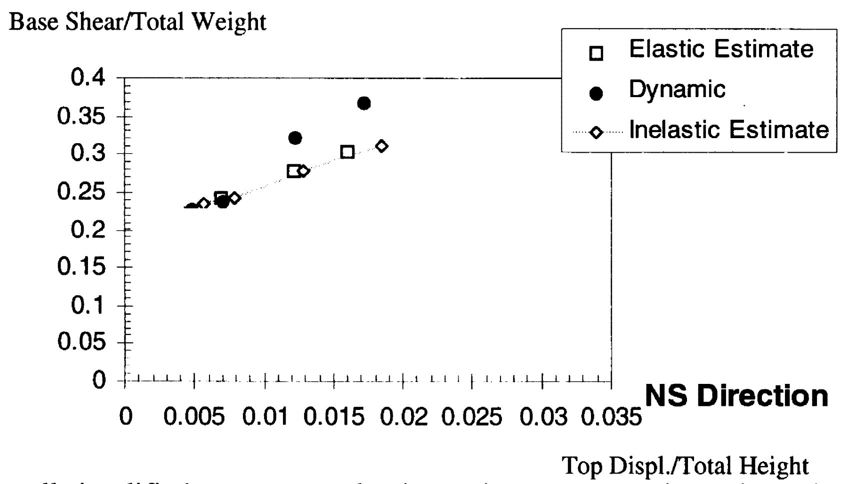

Fig. 8 Overall simplified response evaluation estimates versus dynamic analysis results.

$$DI = \frac{\delta_a - \delta_y}{\delta_u - \delta_y} \frac{1}{\left(1 - \frac{E_h}{4\left(\delta_u - \delta_y\right)F_y}\right)} \tag{5}$$

where δ_a is the maximum experienced deformation; δ_y is the yield deformation capacity; δ_u is the ultimate deformation capacity; F_y is the yield force capacity; and E_h is the cumulative dissipated hysteretic energy.

The fatigue based damage index can be used to qualify the performance of structural elements, stories (or subassemblies), and the overall response of a building. Yield and ultimate capacities were determined using the results from the pushover analyses. Table 2 presents the overall building damage in the NS direction for the five earthquake motion intensities considered. Note that the building is only capable of withstanding and earthquake with a PGA of 0.1g. All other intensities of shaking induce collapse of the structure.

Table 2 Overall damage indices and damage states in the NS direction.

	Maximum Displ. δ_a (in)	Hysteretic Energy E_h (kip-in)	Damage Index DI	Damage State
PGA: 0.1g	1.85	11561	0.08	Repairable
0.2g	2.79	35016	> 1.0	Loss of Building
0.3g	4.01	64760	> 1.0	Loss of Building
0.4 g	7.10	161793	> 1.0	Loss of Building
0.5 g	9.93	316212	> 1.0	Loss of Building

CONCLUSIONS

The seismic evaluation of an existing low-rise RC building was summarized. Five ground motion intensities were considered. The evaluation was carried out using nonlinear time-history analyses, and simplified elastic and inelastic response evaluation methods. Results for the three methods show fairly good agreement in the prediction of the response. However, the simplified methods have the advantage that the evaluation process involves considerably less computational effort. Damage quantification of the building response indicate that the structure can withstand an earthquake with a PGA of 0.1g with repairable damage, but an earthquake with a PGA of 0.2g or greater will collapse the building. The presented example shows different evaluation procedures that can be applied for other building types.

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